GEOTECHNICAL DESIGN REPORT

for the
Construction of Six-Lane Freeway from Airway Road to the Otay Mesa Border Crossing

State Route 905 (Phase I)

11-SD-905
KP 18.5/19.3
11-091801

California Department of Transportation
District 11
San Diego, California

June 2001

Caltrans

State of California — Business Transportation and Housing Agency

MEMORANDUM

To: R.A. Hopkins
Deputy Director - Design

Date: June 25, 2001

File: 11-SD-905
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11-091801

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services – Office of Geotechnical Services
Geotechnical Design Branch-South

Subject: GEOTECHNICAL DESIGN REPORT

In accordance with a request from District 11 Design, enclosed for your consideration is the Geotechnical Design Report, which defines the geotechnical conditions determined or assumed in the development of the geotechnical design recommendations including specific construction criteria for the subject project.

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1. Introduction

The proposed project is Phase I of the freeway connecting Interstates 5 and 805 to the Otay Mesa Port of Entry in southern San Diego County (see *Figure 1 – Vicinity Map*). The Phase I project principally improves capacity via widening of the existing alignment (Interim SR 905) and replacement of an at-grade signalized intersection (Siempre Viva Road) with a grade-separated interchange (overcrossing structure).

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the roadway portions of the project. This report also establishes a geotechnical baseline to be used in assessing the existence and scope of changed conditions. The geotechnical investigation consisted of review of existing reports and geotechnical literature, subsurface exploration (soil borings), laboratory testing and engineering analyses.

This report is intended for use by the project roadway design engineer, construction personnel, bidders and contractors.

2. Existing Facilities and Proposed Improvements

Figure 2 – Site Plan shows the proposed improvements relative to the existing SR 905 (Interim) alignment and local surface streets. Existing SR 905 (Interim) is a four-lane road with paved median and turn lanes to Siempre Viva Road. The proposed improvements consist of the following:

1. Construction of approximately 782 m of six-lane freeway, SR 905, from Sta. "E" 203+00 (KP 18.5) to 210+82 (KP 19.3). This will essentially mirror the alignment and profile of the existing alignment. A minor fill, variable in depth but less than 2 m, will be required to raise the roadbed from approximately Sta. "E" 203+00 to 204+00.
2. Construction of an overcrossing structure for Siempre Viva Rd. Maximum depths of fill range between 6.8 and 7.6 m for the west and east approach embankments, respectively. Approximately 480 m of Siempre Viva Rd., from Sta. "SVR" 97+49 to 102+29, will be raised along its current alignment to cross over mainline 905.
3. Construction of five connector ramps between SR 905 and Siempre Viva Rd. The five ramps consist of the following:
    - **a.** Onramp "SV1" (northeast quadrant of intersection) - westbound Siempre Viva Rd. to northbound SR 905 (construction of this ramp will require a Type I retaining wall to retain a minor cut into an existing slope that descends from an adjoining commercial lot.
    - b. Offramp "SV2" (southeast quadrant) – northbound SR 905 to eastbound Siempre Viva Rd. (fill depths vary from 1 m at Sta. 10+00 to 6.7 m at Sta. 12+70) (construction of this ramp will require three Type I retaining walls to retain embankment fill)

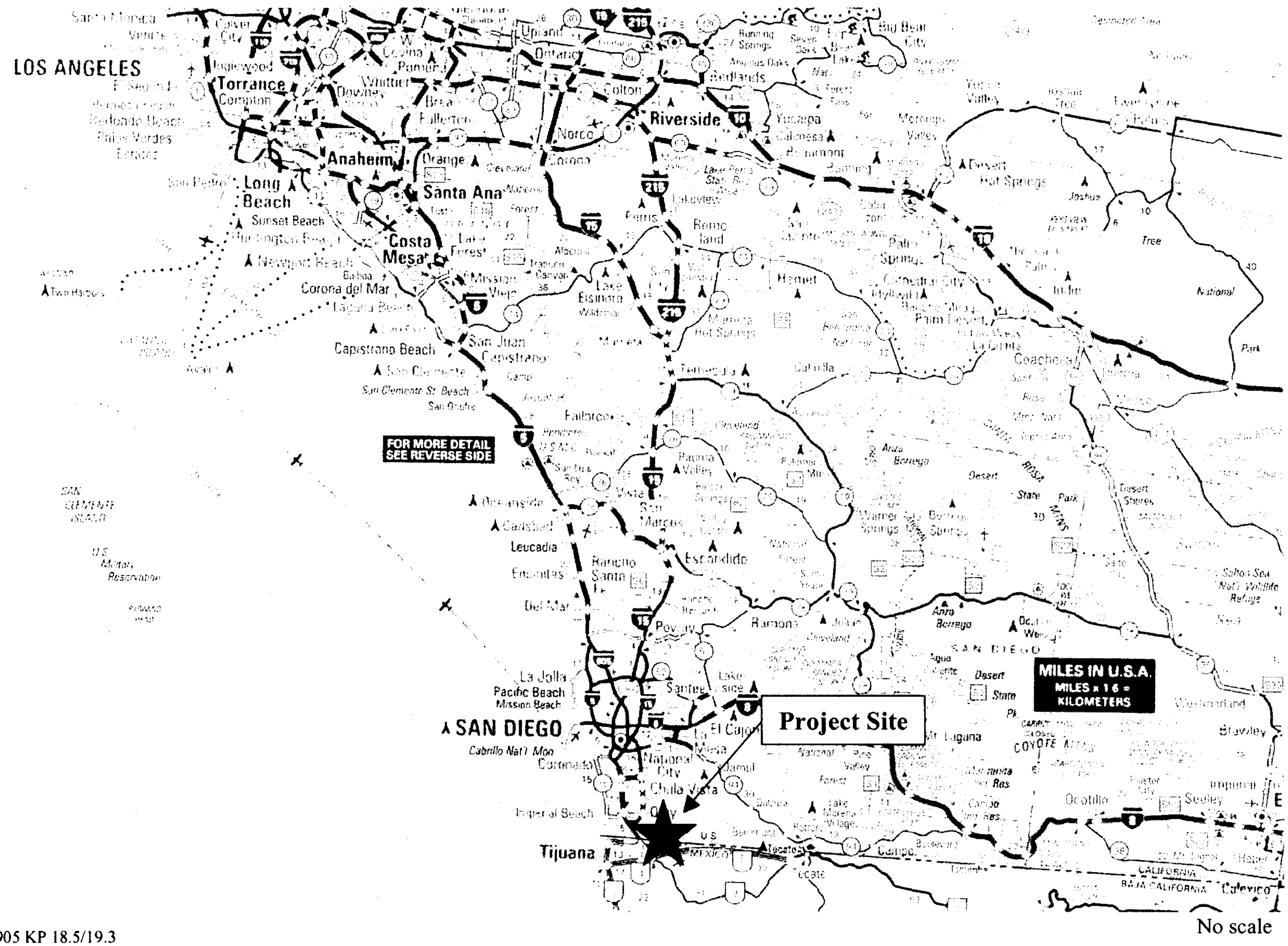

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FIGURE 1 - VICINITY MAP

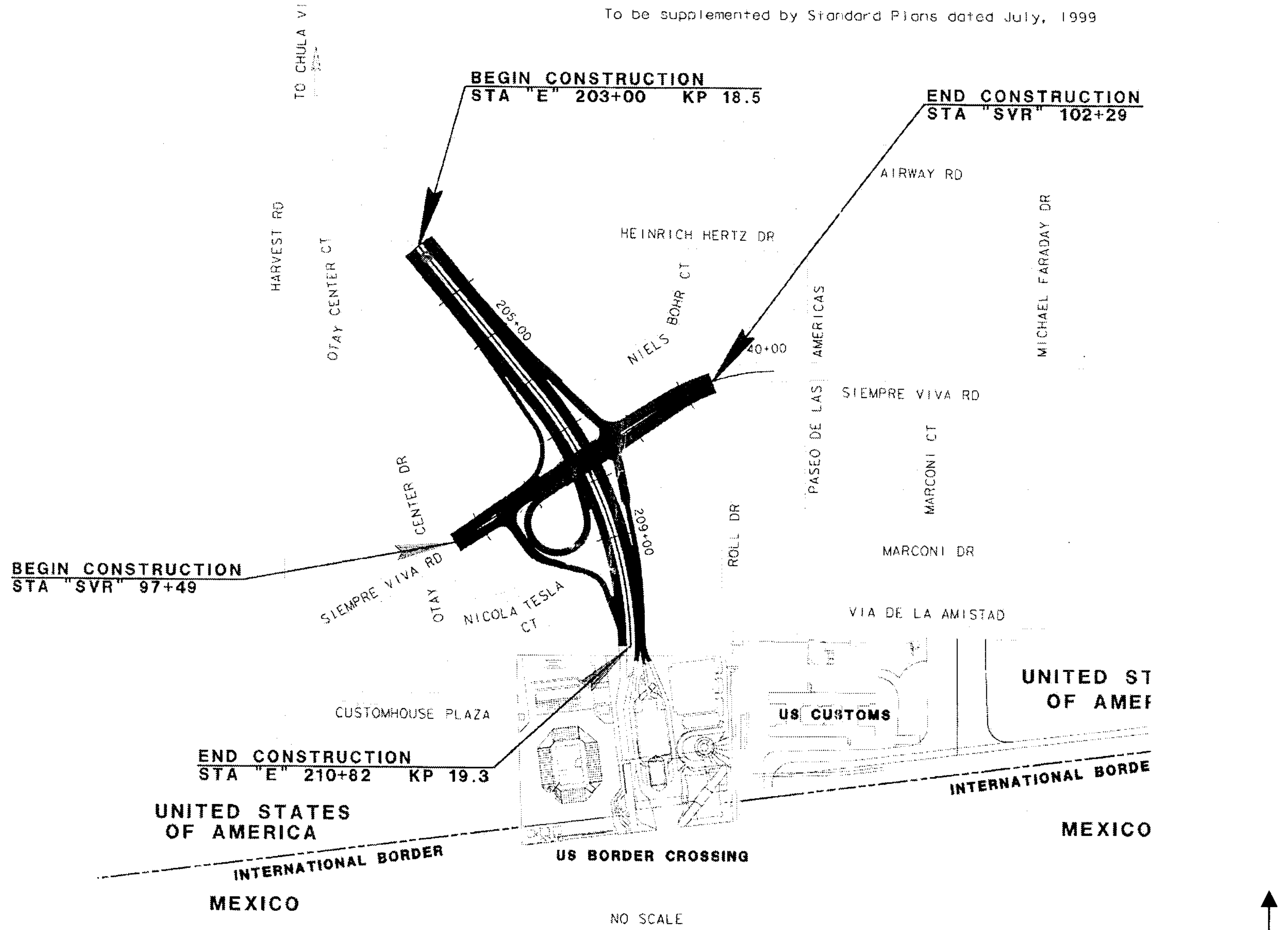

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FIGURE 2 - SITE PLAN

c. Offramp (loop) "SV3" (southwest quadrant) – southbound SR 905 to eastbound Siempre Viva Rd. (fill depths vary up to a maximum of 5 m located at approximately Sta. "SV3" 12+90). A minor cut (1 m or less) is planned at approximately Sta. "SVR" 10+40 to 11+00.
d. Onramp "SV4" (southwest quadrant) – eastbound Siempre Viva Rd. to southbound SR 905 (fill depths vary up to a maximum of 4 m at approximately Sta. "SV4" 12+40)
e. Offramp "SV5" (northwest quadrant) – southbound SR 905 to westbound Siempre Viva Rd. (fill depths vary up to a maximum of 4.4 m at approximately Sta. "SV5" 12+70)

Proposed cut and fill slopes for the project are inclined at 1:2 (v:h).

We observed no pavement distress, which might be suggestive of unstable soils, to existing facilities at the time of our investigation.

3. Pertinent Reports and Investigations

Pertinent reports used in preparation of this document include the following:

1. Preliminary Layout and Profile Sheets for 11-SD-905 Phase I Project, EA 091800.
2. Draft Project Report (DPR), 11-SD-905 KP 9.2/19.3, 11224-093160.
3. "Geologic Reconnaissance and Limited Geotechnical Evaluation for Route 905 EIS/EIR," prepared by Ninyo and Moore Geotechnical and Environmental Sciences Consultants, February 1999.
4. "Preliminary Geological – Geotechnical Investigation I-805/SR-905 From I-805 at Palm Ave. to the International Border Crossing," prepared by Caltrans-RGES, June 27, 1994.
5. "Geological Reconnaissance for Otay Mesa 905 Alignment Study, San Diego, California," prepared by Geocon, Inc., January 1989.
6. "As-Built Grading Plans for Lot 4, Otay International Center," by Rick Engineering, Inc. dated December 7, 1988.
7. "Final Report of Testing and Observation Services During Mass Grading Operations for Otay International Center, Lot 4 and Building Pads 3 Through 14 of Lot 7," prepared by Geocon, Inc., September 1989.

4. Physical Setting

4.1 Climate

Average monthly and annual precipitation and temperature data for Southern California is found on the internet website of the Western Regional Climate Center: http://www.wrcc.dri.edu/summary/climsmsca.html. The Chula Vista weather station (I.D. No. 041758), which is closest to the project site, maintains records since July 1948. The average annual total precipitation for this station is approximately 24 cm, with the heaviest rainfall occuring in the months of January through March. January has the highest average monthly precipiation of approximately 5 cm. The annual average minimum and maximum temperatures for this station are 54.2 and 69.2° F, respectively. The area experiences approximately 340 frost-free days annually.

4.2 Topography and Drainage

The project site is located near the eastern end of Otay Mesa, which is a east-west trending relic wave-cut marine terrace that separates the Otay and Tijuana drainage courses. Pre-development topographical expression in the site vicinity consists of rolling hills of gentle relief. The terrain gradient is approximately 1:6 (vertical:horizontal) toward the southwest. Man-made fills of variable depth and associated with industrial and commercial business parks are located adjacent to the existing SR 905 alignment in the vicinity of the proposed improvements. The elevation of the project site is approximately 165 m above mean sea level.

4.3 Regional Geology and Seismicity

The project site is located within the Peninsular Range geomorphic province, which is characterized by northwest trending mountain ranges separated by subparallel fault zones. In general, the Peninsular Ranges are underlain by Jurassic metavolcanic and metasedimentary rocks and by Cretaceous igneous rocks of the southern California batholith.

The Peninsular Range province can be divided into two minor physiographic provinces: the Coastal Plain and the Interior Upland. The project site is located within the Coastal Plain, however, the transition to the Interior Upland is located approximately 5 km to the east. The Coastal Plain is dominated by rocks of the San Diego Embayment, consisting of Upper Cretaceous, Tertiary, and Quaternary sediments. These sediments are principally comprised of indurated to friable pebble, cobble and boulder conglomerates, sandstones and claystones. Quaternary sea advances and retreats cut the tertiary sediments into a series of terraces that parallel the present coastline. Subsequently, drainage courses dissected the wave-cut terraces to form this vicinity's characteristic mesa topography.

Figure 3–Regional Fault Map shows active and potentially active faults within approximately 100 km of the project site. Faults indicated in red and orange are considered "active" faults. Those shown in red indicate surface displacement within historic times (past 200 years), whereas those indicated in orange have shown movement during Holocene epoch (past 10,000 years). "Potentially active" faults, shown in green and purple, have shown evidence of Quaternary displacement (during the past 700,000 and 1,600,000 years, respectively).

The following table summarizes parameters of known "active" and "potentially active" faults within a 100-km radius of the project site, listed in order of increasing distance from the project site, including approximate site-to-source distances and maximum credible earthquake magnitudes. Due to its proximity to the project site, the Newport-

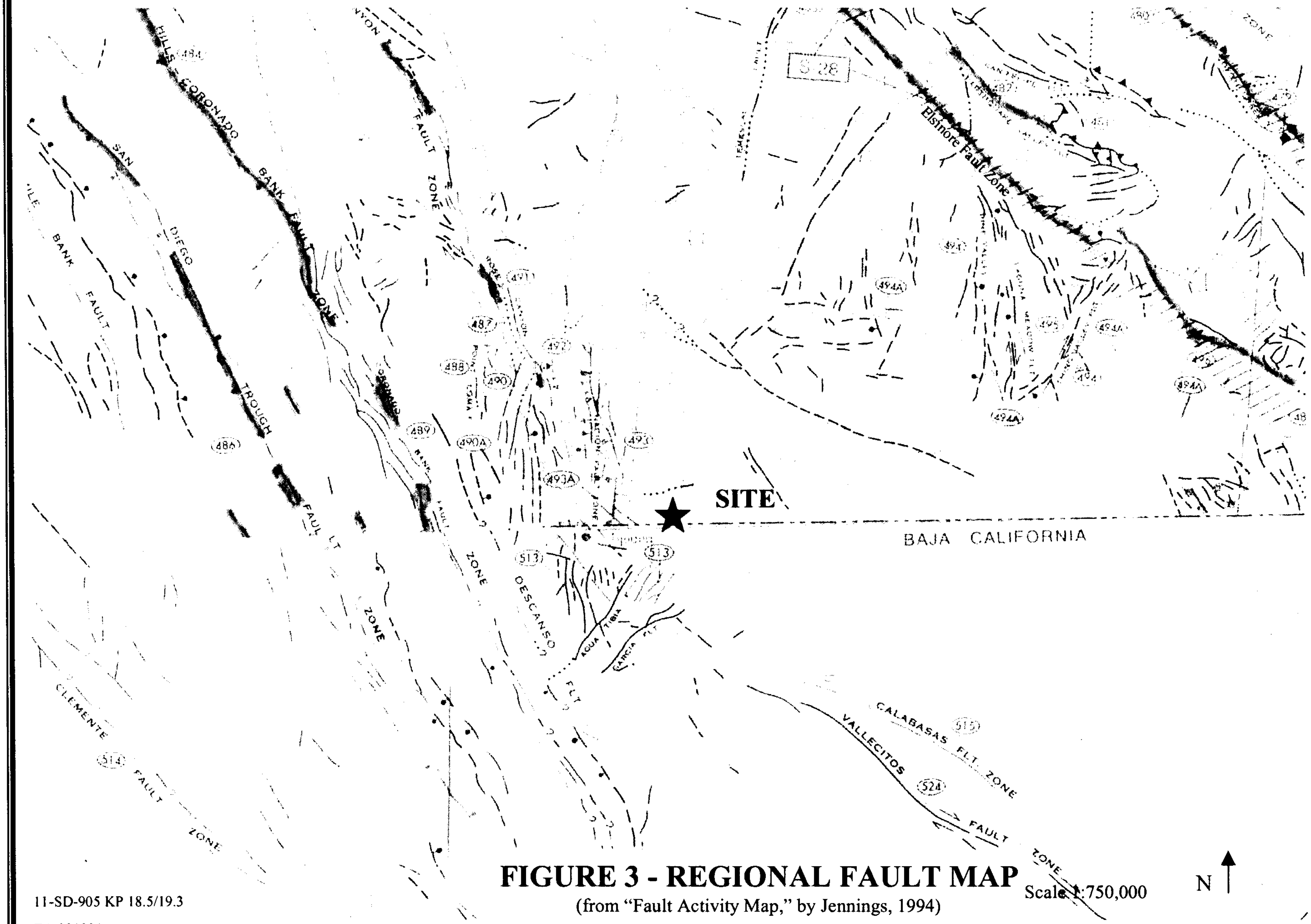

FIGURE 3 - REGIONAL FAULT MAP

(from "Fault Activity Map," by Jennings, 1994)

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Inglewood Rose Canyon Fault dominates seismicity in the region as other active faults of identical magnitude are located significantly further from the site. The Rose Canyon Fault exhibits a strike-slip style of faulting.

The La Nacion Fault Zone is the closest known Quaternary fault to the project site (approximately 9 km to the west) and was included in the 1990 Caltrans Seismic Hazard Map (Mualchin). The fault was subsequently omitted from the map in the 1996 revision. The County of San Diego considers this fault to be "active," claiming it has shown Holocene displacement. Correspondingly, there is some disagreement as to the activity of this fault and its ability to generate a maximum credible event. Nonetheless, it has been considered here. The La Nacion Fault exhibits a dip-slip style of faulting.

Fault Parameters (Mualchin, 1990 and 1996)

Fault Name	Approximate Distance (km)	Maximum Credible Magnitude
La Nacion Fault Zone	9	6.5
Newport Inglewood - Rose Canyon Fault Zone	14	7.0
Point Loma	30	6.5
Palos Verdes Hills-Coronado Bank	31	7.75
San Diego Trough	48	7.5
Whittier-Elsinore Fault	66	7.5
San Clemente	92	7.25

4.4 Soil Survey Mapping

When the United States Department of Agriculture-Soil Conservation Service (USDA-SCS) mapped this area in 1973, landuse in this area was devoted principally for agricultural purposes and as such, development had not significantly altered the natural soil profile. Since 1973, the immediate area near Siempre Viva Rd. has seen a significant degree of transportation and commercial development including construction of Interim SR 905 and adjoining business park properties on both sides. One limitation of using USDA-SCS soil surveys to evaluate geotechnical conditions is that they only evaluate the upper 1.5 m of the soil profile and do not describe soils at greater depth.

The soil unit mapped for this area is Diablo Clay (Dac), which is a weathered soil horizon comprised of dark gray clay derived from soft, calcareous sandstone and shale (Otay Formation). We did not encounter this specific horizon during our subsurface investigation, which may suggest that this material has been removed or reworked as a result of grading activities in the area. Surface water runoff for the Diablo clay is characterized as slow to medium and the erosion hazard slight to moderate. Diablo clay is assigned to the *hydrologic soil group D*, which indicates a soil having very slow infiltration rate when wetted. This parameter is provided for the District NPDES unit to use in assessing erosion potential. We anticipate that the underlying formational material (Otay) will be exposed to some degree in cuts along Ramp SV-1 and reason that it belongs to the same hydrologic group (D) due to high percentages of silt and fine sand particles (see laboratory test data). Erodibility of cut-slopes is discussed further in Section 7.3.1.1.

5. Exploration

A geotechnical subsurface investigation was conducted to establish a vertical soil profile across the site and to determine soil types and their engineering characteristics within the proposed project limits. These characteristics have been used to anticipate soil behavior during and after construction of the proposed improvements. The subsurface investigation was comprised of drilling with Standard Penetration Testing (SPT) and undisturbed tube sampling.

5.1 Drilling and Sampling

A Caltrans Mobile B-47 drill rig was used to advance five mud-rotary borings (Borings 600-1 through 600-5) in the project vicinity in late May 2000. The mud-rotary technique uses a drilling fluid thickened with bentonite powder or polymer to stabilize the walls of the borehole and to bring cuttings to the surface. Locations of the borings are shown on *Figure 4* and annotated on the appended boring logs. Depth of borings varied from 9.6 to 14.2 m with the deepest borings adjacent to the proposed structure approach embankments. Surface elevations of the borings varied from 162.53 to 166.04 across the site.

Disturbed samples were obtained during Standard Penetration Testing (SPT). Relatively undisturbed tube samples were obtained with a 63.5-mm dia.(outer) Modified California Sampler. The Modified California Sampler is comprised of a 63.5 mm diamter split barrel that holds six 152.4 mm brass sleeves. Due to the hardness of the formational material, these samples were obtained by driving the California sampler ahead of the drilling bit. The undisturbed sampling interval varied between boreholes and with depth (see boring logs). The sampling interval for SPT sampling was generally 1.5 m. SPT blow counts were used to determine soil consistency or relative density. Samples were trimmed flush with the sleeve's ends; these ends were covered with plastic caps and the resulting seams were taped.

SPT and undisturbed samples were sent to the Office of Structural Foundations Geotechnical laboratory (Sacramento) for laboratory testing. Untested samples are currently stored at the District 11 Materials Laboratory.

Boring logs are included in Appendix B.

5.2 Instrumentation

Boring 600-1, adjacent to loop ramp SV-3 and the existing storm water detention basin, was converted to an observation well to define perched ground water conditions. The well consists of 50-mm diamter slotted PVC pipe backfilled with sand filter to within 1.5 m of the ground surface, followed by solid wall PVC pipe and bentonite plug to the ground surface.

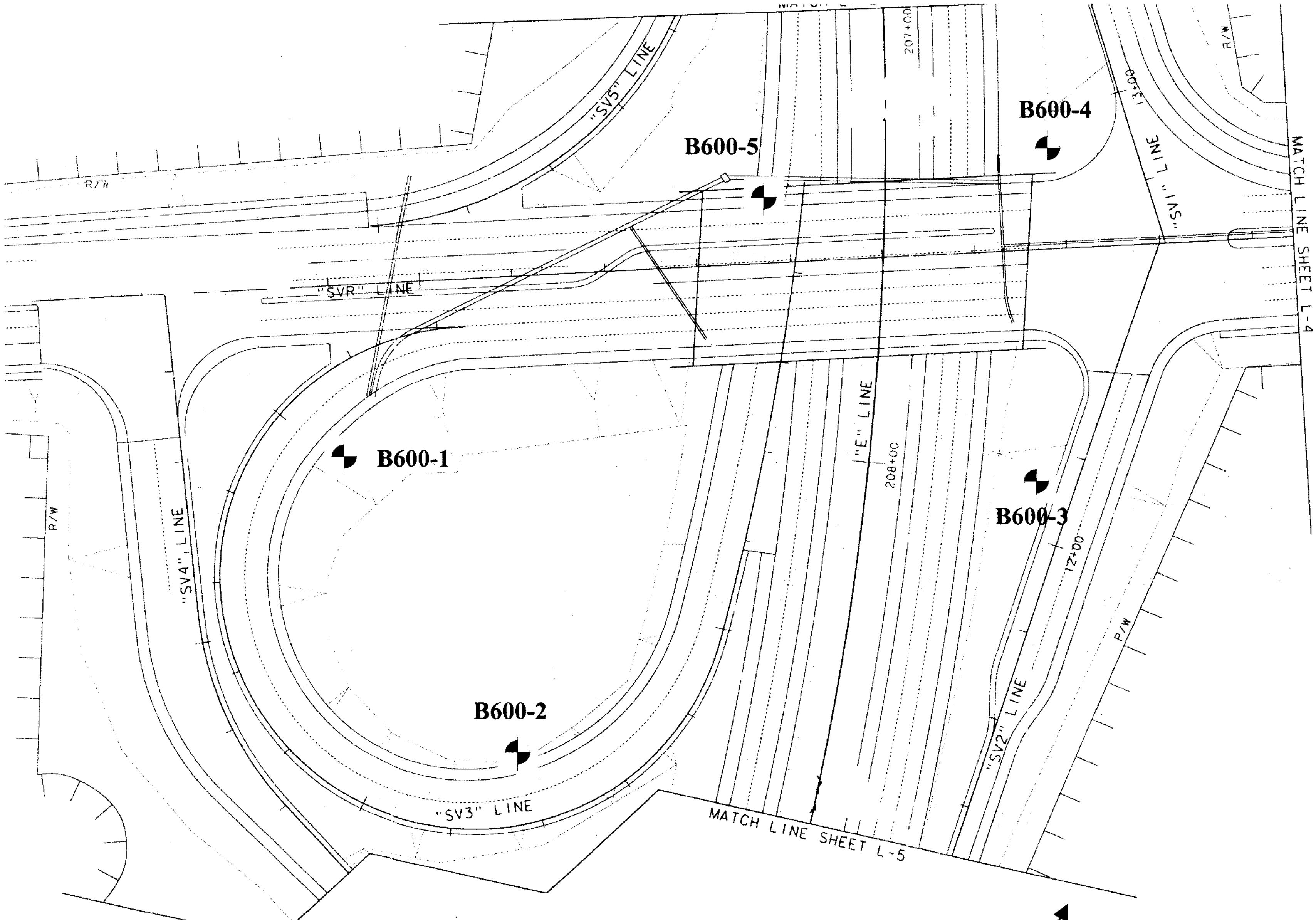

FIGURE 4 - BORING LOCATION MAP

(base map is Layout Sheet L-3)

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6. Geotechnical Laboratory Testing

Laboratory testing was performed in accordance with California Test Method (CTM) and American Society of Testing Materials (ASTM) standards at Translab in Sacramento. The table below summarizes the testing program used for this investigation. Test results are included in Appendix C.

Laboratory Testing Schedule

Test	Standard	Number of Tests
Moisture and In-situ Density	ASTM D 2937-83	20
Mechanical Analysis	CTM 201, 202, 203	7
Atterberg Limits	CTM 204	3
Specific Gravity	CTM 209	7
Consolidation	CTM 219	5
Direct Shear	ASTM D 3080-90	3

7. Geotechnical Conditions

7.1 Site Geology

The project site is located on Otay Mesa, which is situated near the inland edge of a sediment filled basin (the San Diego Embayment). Volcanic basement rock surfaces just a few kilometers east of the site. The surface of Otay Mesa generally represents an erosional remnant of Tertiary sedimentary beds cut by Quaternary sea advances. These beds are believed to be comprised of both marine and terrestrial sediments.

7.1.1 Lithology

The project alignment is wholly underlain by the oligocene-aged Otay Formation (Tm) rock unit *(see Figure 5-Geologic Map)*. The Otay Formation is described in the literature as light-gray and light-brown, poorly indurated, massive sandstone and claystone (Kennedy and Tan, 1977). Sandstone is typically poorly cemented and claystone is partially bentonitic (expansive) in composition. The thickness of the Otay Formation at this location is at least 15 m, which was the maximum depth explored during our subsurface investigation.

7.1.2 Structure

The Otay Formation appears to have a minor bedding dip (5°) towards the southwest in the general vicinity of the project site, which essentially mirrors the topographic descent in this direction. No other structural characteristics are notable with respect to geotechnical design or construction.

7.2 Subsurface Soil Conditions

Soil borings revealed little spatial variation in surface soils within the project area. Soil

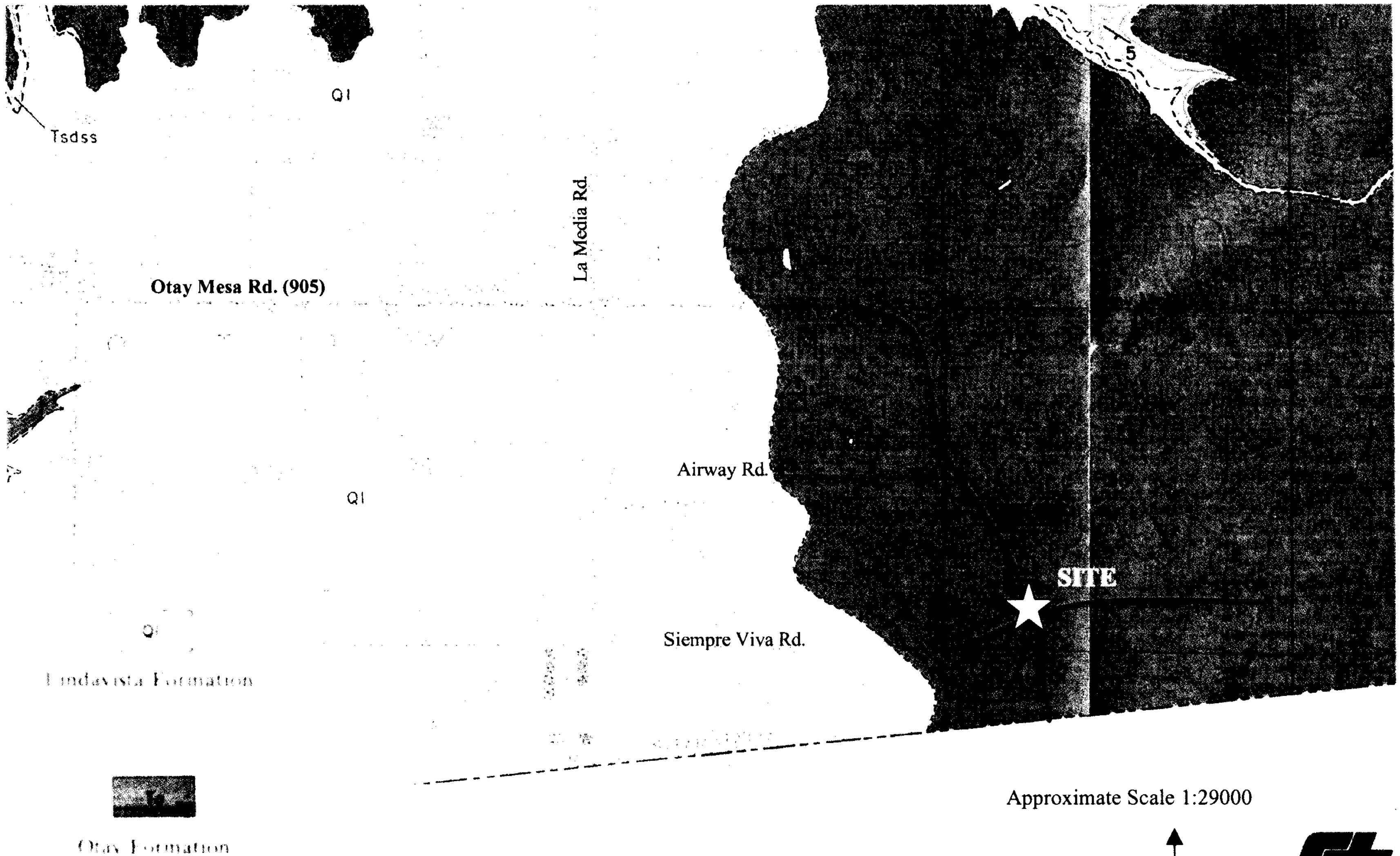

FIGURE 5 - GEOLOGIC MAP

(Reference: *Geology of national City, Imperial Beach and Otay Mesa Quadrangles, Southern San Diego Metropolitan Area, California*, by Kennedy and Tan, 1977) in the upper two meters generally consists of moist, light-gray to light-brown, medium dense to dense, clayey sandstone. Laboratory testing confirms that site soils have been preloaded as a result of additional deposition and subsequent erosion of overlying sediment during geologic time. This stress history has rendered the soil in an "overconsolidated" state, meaning that the soil particles have been compressed to a denser configuration and correspondingly, will compress relatively less when loaded by wall footings or embankment fill. The sandstone is weakly cemented and fines possess medium to high plasticity.

Due to its proximity to the ground surface, the upper two meters has undergone some weathering processes, which has acted to loosen the soils somewhat. SPT blow counts per 0.3-m in this upper zone varies between 15 and 41. Soil type remains the same while density increases significantly with depth below this upper zone to a maximum depth explored of 15 m. Blow counts per 0.3-m at depths greater than 2 m range from 28 to more than 100.

7.3 Water

7.3.1 Surface Water

The most notable drainage feature within the project area is an approximately 3-m deep detention basin located within the proposed SV-3 loop ramp. A culvert carries runoff from watershed areas north of Siempre Viva Road (SVR) via culvert to the detention basin. This culvert will be relocated as part of the proposed improvements.

7.3.1.1 Erosion

Existing slopes within the project area are landscaped with no visible signs of erosion. Erosion potential for exposed temporary and permanent cut slopes is moderate. For the purposes of estimating soil loss (by others), a soil erodibility factor, *k*, of 0.41 can be conservatively assigned to soils exposed in cuts and fills (reference Goldman et al). This value can be applied to fill slopes provided the compacted soils are similar in gradation to the native soils.

7.3.2 Ground Water

The regional ground water table was not encountered in any of the borings for this investigation. It is located at significant depth relative to the proposed construction and consequently, will not impact the project.

Boring B600-1, located approximately 5 m from the detention basin adjacent to loop ramp SV-3, was developed into a standpipe piezometer to assess perched water conditions underlying the project site. The water level in this piezometer read approximately 3 m below ground surface (el 159.6+/-) in January 2001. This level approximately matches the base level of the nearby detention basin and may not be continuous across the entire site. The Contractor may encounter some perched water seepage in cuts as a result of regular and relatively heavy landscape irrigation within the project limits.

7.4 Project Seismicity

A deterministic hazard analysis was performed using the California Seismic Hazard Map (Mualchin, 1990 and 1996).

7.4.1 Ground Rupture

There are no known faults that cross the project site and thus, no potential for ground rupture based on current fault information.

7.4.2 Shaking

The 1990 and 1996 (revision) of the California Seismic Hazard Map (by Mualchin) and was used to determine the peak ground acceleration that the project site would experience due to a maximum credible earthquake.

Figure 6 (1996 map) shows the relationships of faults capable of generating a maximum credible event and contours of peak bedrock acceleration with respect to the project site. The dominant fault for the site is the Rose Canyon Fault, which is abbreviated as NIW (Newport Inglewood Fault), as it is regarded as a southern extension of this latter fault. A maximum credible event (MCE) magnitude of 7.0 is assigned to the Rose Canyon Fault. Based on the 1996 map, which does not include the La Nacion Fault, the peak bedrock acceleration for the site is approximately *0.30 g*. Based on the 1990 map, the peak bedrock acceleration is increased slightly to 0.37 g, due to consideration of a 6.5 event on the La Nacion Fault.

8. Geotechnical Analysis and Design

8.1 Seismic Analysis

8.1.1 Seismic Parameter Selection

A pseudo-static coefficient of 0.15 was used in computing global stability of the Type 1 Retaining Walls along ramps SV-1 and SV-2. (This number generally is accepted as the standard of practice for the Southern California area as it applies to slope stability.) A value of 0.15 for the pseudo-static coefficient is also in agreement with published guidelines recommending 1/3 to ½ of the peak bedrock acceleration values (see Section 7.4.2).

The soil conditions at the project site do not warrant a site-specific analysis to further define estimates of ground motion parameters.

8.1.2 Embankment Stability

All embankment slopes within the project area are assumed to be globally stable under seismic loading assuming a 1:2 (v:h) slope ratio and a pseudo-static coefficient of 0.15. Dynamic slope stability analyses of embankments were not performed. Use of slopes steeper than 1:2 (v:h) may present maintenance issues with regards to landscaping. We

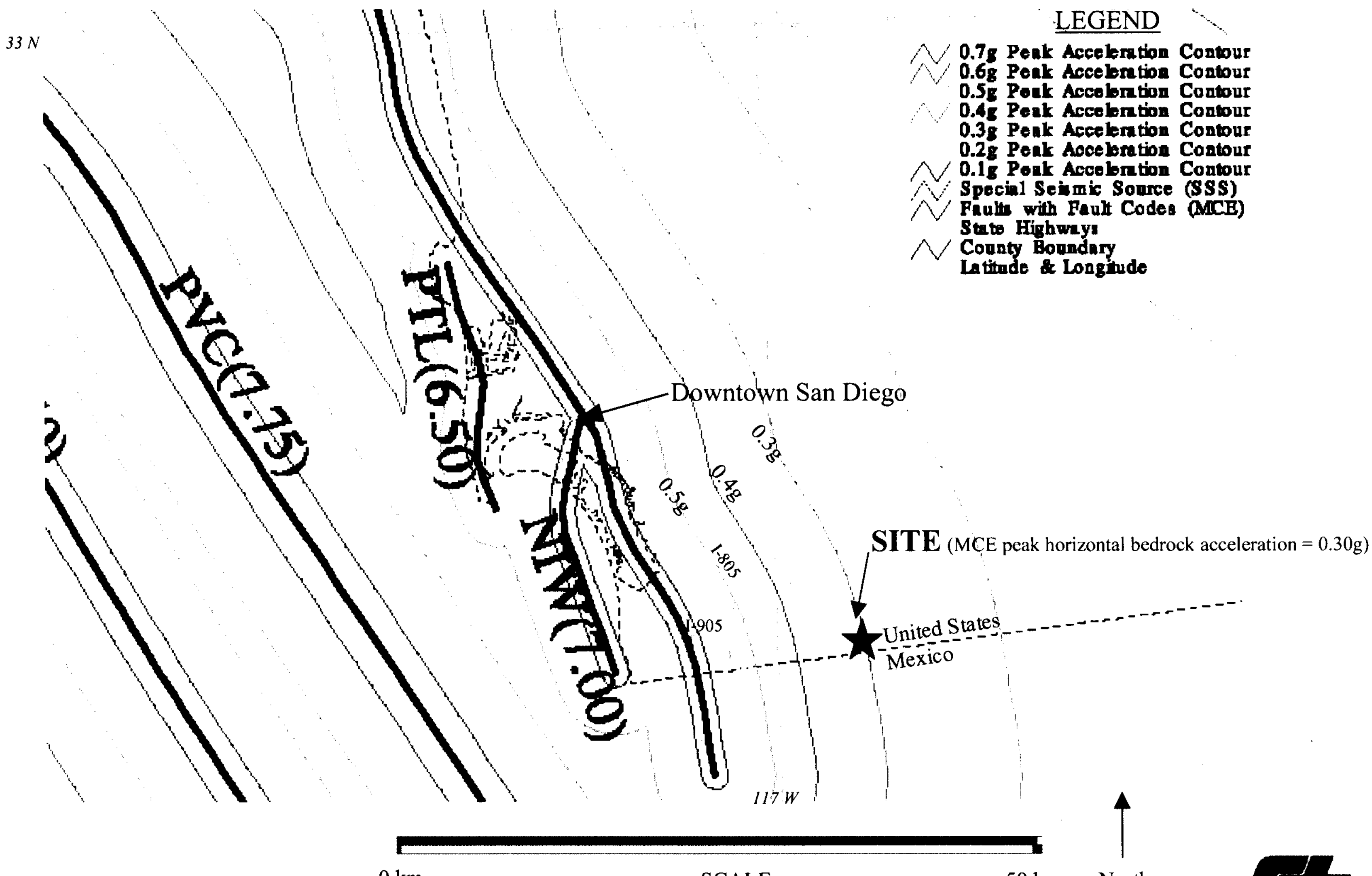

FIGURE 6 - SEISMIC HAZARD MAP

(from *California Seismic Hazard Map 1996* by Lalliana Mualchin)

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therefore recommended consultation with our office and District Landscape Architecture department if slopes steeper than 1:2 (v:h) are proposed.

Site soils are not liquefiable due to high fines content, overconsolidated nature of the formational soils and deep regional ground water table. Mitigation measures to prevent liquefaction are not warranted.

8.2 Cuts and Excavations

8.2.1 Rippability

Site soils can be excavated with conventional earthmoving equipment. The majority of the cut on the project is planned along the SV-1 onramp and is predominantly in formational material. The maximum depth of cut in this area will be approximately 3 m. Blasting or other special means of excavation will not be necessary.

8.2.2 Grading Factors

A grading factor of 0.93 is presented in the Materials report for the project. This number is based on soils exploration in the upper 1.5 m, however, is is also considered applicable at slightly greater depth due to consistency of the formational soils with depth.

8.3 Embankments

Based on profile grade sheets provided by District Design, the maximum proposed depth of fill for the approach embankments to the Siempre Viva overcrossing is approximately 7.6 m. Consolidation test data for undisturbed samples and SPT blow counts in borings B600-4 and B600-5 were used to evaluate the potential for post-construction consolidation settlement. Soil with blow counts exceeding 50/0.3 m were considered incompressible for engineering purposes and not included in the analysis. Boring logs indicate that soils below approximately 4 m have blow counts exceeding this criterion. Our calculations show that a maximum of 50 mm of post-construction settlement can be expected over a period of approximately 30 days, however, there is a strong likelihood that a portion of the computed post-construction settlement will actually occur during construction. Settlement calculations are included in Appendix F.

Embankments associated with ramp construction have a maximum fill height of approximately 4.4 m. The maximum expected post-construction settlement is 30 mm, however, it is likely to be less than this amount for the reasons cited above. We do not see a need for a settlement waiting period prior to paving for these embankments.

Proposed embankment slopes inclined at 1:2 (v:h) slope ratio are globally and surficially stable under static and dynamic loading conditions. Detailed stability analyses for the embankments were not performed due to high strength of the formational foundation soils and relatively high strength characteristics of engineered fill placed at 90% relative compaction.

All embankments for the project should be constructed in accordance with Sections 19-5 and 19-6 of the 1999 Standard Specifications.

8.4 Earth Retaining Systems

<u>Type I Retaining Wall, SV-1 Ramp (Sta. 12+43-Sta. 12+91)</u>

This standard plan wall is approximately 48 m in length, has a maximum height of 2.4 m and will retain both formational and fill soils. The wall was evaluated using a Case II loading condition (unlimited 1:2 (v:h) slope above the wall).

Bearing capacity calculations assume a 4.8 m-high wall, which was the original design height. The wall has been realigned since, resulting in a reduced wall height of 2.4 m. Foundation soils will provide sufficient bearing capacity. A net allowable bearing capacity of 300 kPa was computed based on a 0.4-m embedment depth, a 1.3-m footing width, and a safety factor of 3.0. This far exceeds the maximum toe pressure of 70 kPa for the maximum proposed wall height. Calculations for bearing capacity are included in Appendix F. Minimum factor of safety criteria (1.5) are met for overturning and sliding as well.

Global stability of the retaining wall was assessed using the computer program XSTABL, version 5. Janbu's Simplified Method of Slices (limit equilibrium theory) was used in computing factors of safety for both static and dynamic (earthquake) loading at Station 12+60. Figures 7 and 8, below, show site geometry and soil parameters used in the analyses and computed factors of safety. The minimum required factors of safety for static and dynamic loading are 1.5 and 1.1, respectively. These minimum values are easily exceeded. The Xstabl output files are included in Appendix F.

Shear strength parameters for the native soil were computed using averaged results of direct shear tests on driven tube samples obtained during our investigation. A friction angle of 30° for compacted fill was conservatively assumed.

A surcharge footing load of 96 kPa for the adjacent commercial building was found to have negligible effect on the minimum factor of safety. Based on the daylight location of the failure surface corresponding to the minimum factor of safety, stability of the commercial building should not be adversely impacted by the proposed wall and final slope configuration.

A temporary excavation will be required for the construction of the Type I retaining wall along the SV-1 onramp. Construction personnel prior to excavation should consider the position of the existing building and masonry wall outside of the R/W relative to the daylight point of the temporary cut. Based on cross-sections furnished to our office by District 11-Design, the building edge is located approximately 6 m from the top of slope.

At this time of this writing, we do not anticipate any adverse effects on the temporary stability of this cut. Based on as-built grading plans (Rick Engineering, 1988), we anticipate that formational soils overlain by engineered fill of variable thickness will be encountered.

Figure 7 – Global Stability (static)
SV-1 Ramp

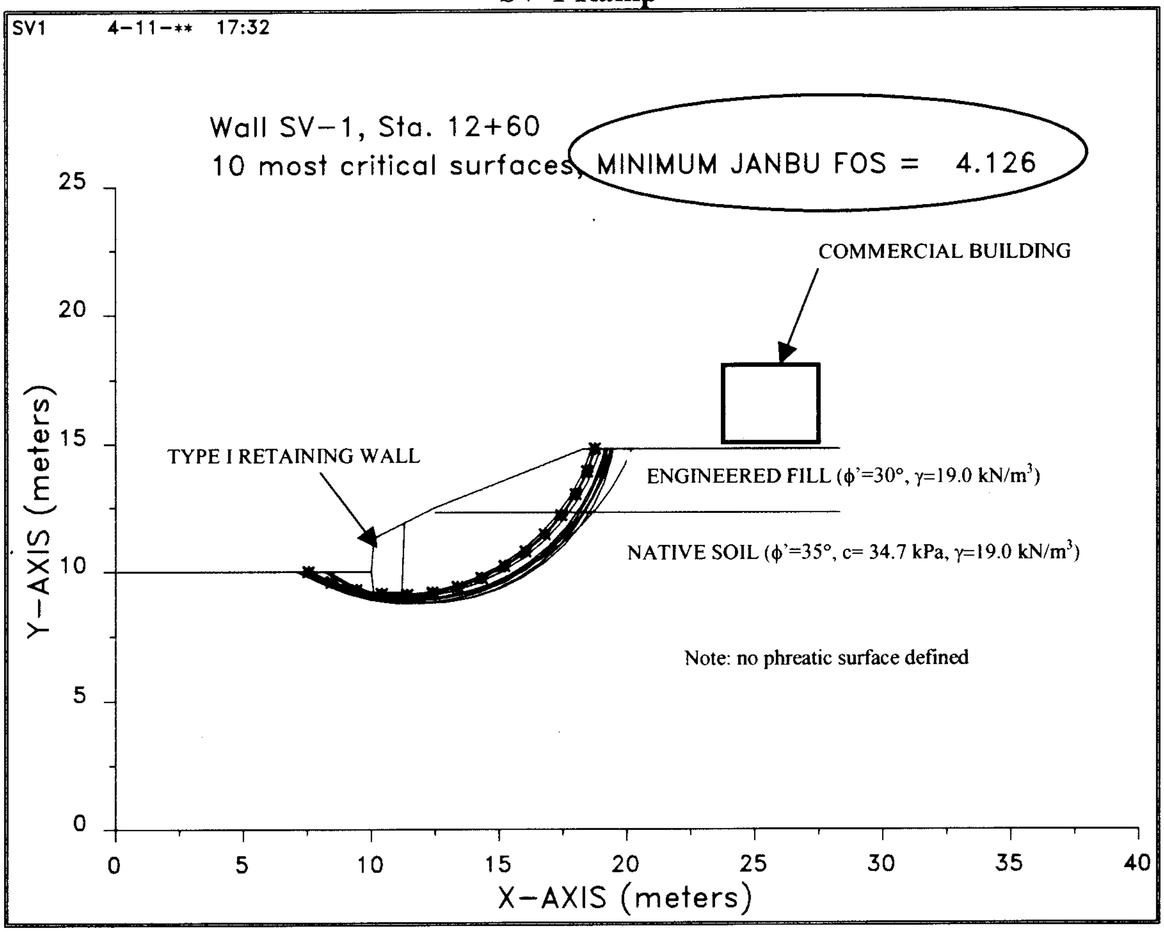
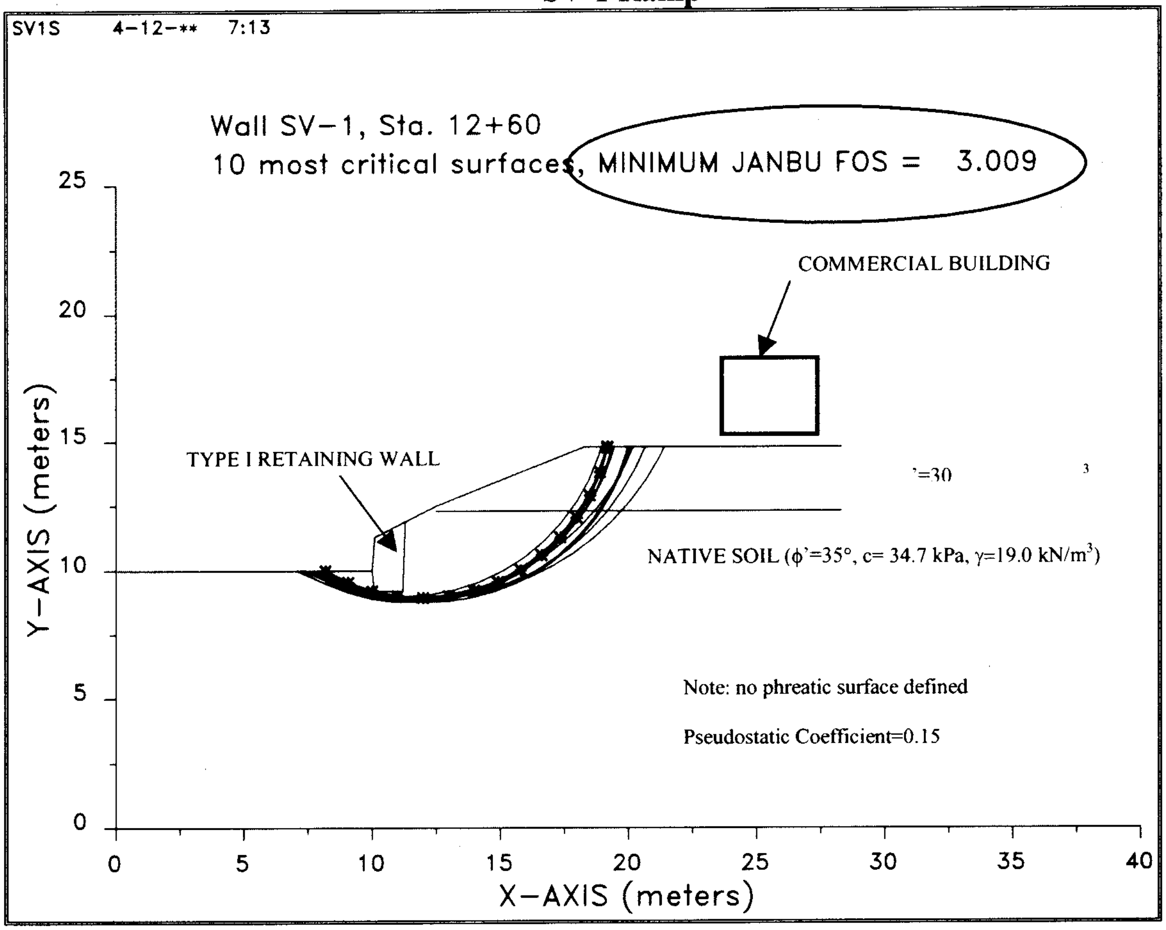


Figure 8 – Global Stability (pseudostatic case)
SV-1 Ramp



Type-1 Retaining Walls, SV-2 Ramp

There are three sections of Type-1 wall planned along the SV-2 ramp. They vary in length from 13.5 m to 60 m. The following table summarizes their starting and ending stations, maximum heights, design loading conditions and maximum applied toe pressure:

Wall Designation	Starting Sta. (Wall LOL)	Ending Sta. (Wall LOL)	Maximum Wall Height (m)	Design Loading Conditions	Maximum Applied Toe Pressure (kPa)
SV-2a	10+87.20 (SV-2 align.)	11+25.60 (SV-2 align.)	2.4	Case I	105
SV-2b	11+70 (SV-2 align.)	12+30 (SV-2 align.)	3.0	Case I	120
SVR_SV2	13+65 (sv2rep1 align.)	13+78.48 (sv2rep1 align.)	3.6	Case I	135

A Case I loading condition (level ground above the wall with a 11.5 kPa live traffic surcharge) was used to determine a maximum toe pressure from the Standard Plan Sheet B3-1.

Site soils provide allowable bearing capacity that exceeds the maximum toe pressures for each wall. Long-term settlement of the walls is expected to be minor and tolerable as the formational foundation soils are dense to very dense.

Wall SVR_SV2 (total length approximately 13.5 m) will be founded in compacted fill placed as part of the new ramp and approach embankment construction. There will be a short descending slope in front of the wall inclined at a slope ratio of 1:2 (v:h). Provided this fill is placed at 90% relative compaction and conforms to Caltrans Standard Specifications Section 19-Earthwork, allowable bearing capacity should excced 135 kPa with less than ½ inch of maximum total settlement. A minimum horizontal distance of 2.4 m, measured from the front face of the base of footing to the descending slope surface, should be provided in the wall design to protect the footing from potential erosion and undermining.

<u>Type-1 Retaining Walls, Siempre Viva Rd</u>

Figures 9 and 10 (below) show the planned culvert headwall and upslope retaining wall in plan and section views, respectively. The upslope retaining wall, is approximately 20 m in length and reaches a maximum height of 3.0 m.

Figure 9 – Retaining Walls Plan View SVR line (northwest quadrant of project)

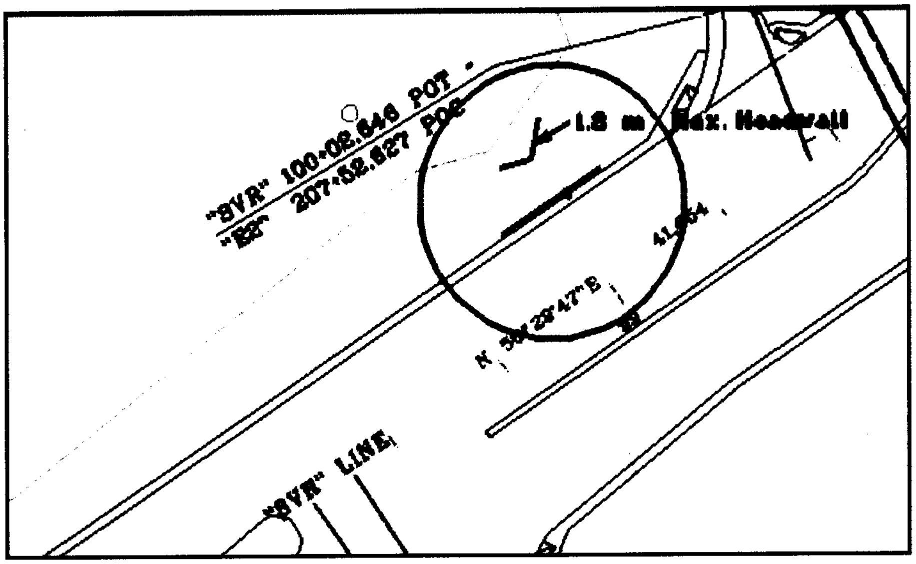
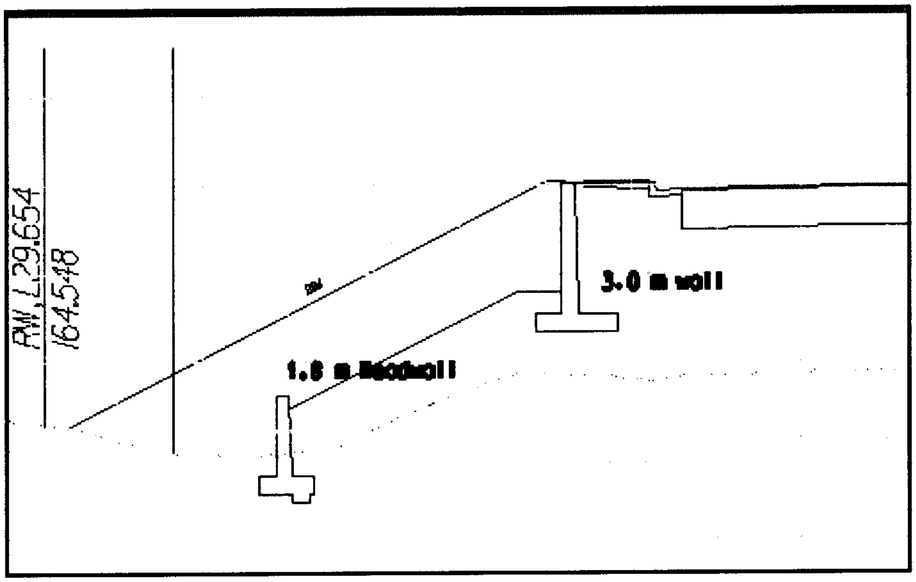


Figure 10 – Retaining Walls Section View SVR Line (Sta. 99+00)



As wall layout was developed after our subsurface investigation, we did not plan a boring at this location. The closest boring is B600-5, which is located approximately 75 m away (towards the overcrossing structure).

One of these walls is a headwall for a new cross culvert that will be constructed at approximately Sta. 99+00 of Siempre Viva Rd (SVR). The maximum height of this headwall is 1.8 m. A relatively short fill slope inclined at 1:2 (vertical:horizontal) will ascend to the upslope retaining wall (Case II loading). Based on the above cross section, we expect that the wall will bear on the formational material encountered in Boring B600-5. The allowable bearing capacity will exceed the maximum applied toe pressure of 90 kPa.

The upslope Type-I retaining wall will likely bear on up to 1 m of new compacted fill. The existing pavement of Siempre Viva Rd. will likely be removed and expose an existing embankment fill that appears to be less than 2 m thick. Fill near the edge of the embankment appears loose based on visual inspection at the ground surface. We therefore recommend that structural backfill be placed for a depth of 1 m below the footing base and a total width of 1.5 times the footing width in accordance with Section 19-3.06 of the Standard Specifications. This will provide an allowable bearing capacity exceeding 120 kPa (Case I loading condition) using a safety factor of 3.0. The wall also meets minimum acceptable factors of safety against sliding and overturning (F.S. =1.5).

Global stability of the two retaining walls was evaluated to be sufficient based on proposed slope inclinations (no steeper than 1:2), no unusual surcharges and relatively high shear strength characteristics of engineered fill. Additional analyses and/or computations for slope stability were not performed.

8.5 Expansive Soils

Expansion potential of near-surface soils was evaluated based on laboratory test data for Atterberg limits, sieve analysis and consolidation tests. Atterberg limits indicate soils with low to medium expansion potential (Miller and Nelson, 1992). Values of plasticity index varied between 9 and 31 with a mean value of 21. One of the consolidation tests indicated minor swelling of the sample upon wetting.

Moisture contents beneath the existing structural section on Interim SR-905 and Siempre Viva Road have likely increased since their original construction and stabilized in response to the constant landscape irrigation throughout this area. Due to relatively high in-situ moisture contents and relatively low clay-size fraction (generally less than 15 percent), we do not expect these soils to swell significantly after the new facilities are constructed. Provided that foundation treatment recommendations for the roadbed subgrade presented in the project Materials Report are followed (scarification/remove and recompact in the upper 0.9 m), the native subgrade soils will be remolded, which will likely act to decrease their swell potential. Wall bearing pressures are sufficient to resist relatively minor swelling pressures that would form in response to moisture increases in the foundation soils.

We did not observe any expansive-soils related distress to existing pavements or walls at the time of our investigation.

9. Corrosion Studies

Corrosion potential of wall foundation and backfill soils was assessed by the Translab Corrosion Technology Branch in Sacramento at the request of Geotechnical Design Branch-South. Corrosion test data (pH, resistivity, chloride and sulfate content) presented in the project Materials Report was furnished to the Corrosion Technology Branch to assist them in developing their recommendations. Our office did not perform any additional corrosion testing.

Recommendations to mitigate against corrosion for all retaining walls on the project are attached as Appendix E.

10. Construction Considerations

Construction personnel should construct and monitor settlement platforms on the approach embankments of the overcrossing structure to ensure completion of 90% primary consolidation prior to construction of bridge abutments and paving. Estimated waiting period for settlement is 30 days.

The Contractor should consider curtailing or reducing slope irrigation in the area of the retaining wall on the SV-1 ramp (northeast quadrant) prior to excavation to limit adverse ground water effects on cut stability.

All wall footing excavations should be inspected by Construction personnel to ensure complete removal of deleterious materials (e.g. organics, bentonitic (expansive) clay seams) that may have gone unnoticed during the geotechnical investigation.

We are not aware of any hazardous waste or material that presently exists within the project limits nor did we encounter hazardous material during the course of our investigation.